Kansas Citys, Missouri and Kansas
Flood Damage Reduction Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Interim Feasibility Report

Chapter A-14

STRUCTURAL ANALYSIS
FAIRFAX-JERSEY CREEK
(BPU FLOODWALL)

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A-14.1 INTRODUCTION

Due to a lack of information, several critical assumptions were made in the analysis and recommendations put forth in the Existing Conditions analysis. Recognizing the impact of these assumptions, extensive field testing was performed to obtain additional information to be used to more accurately define a level of reliability for the Fairfax-Jersey Creek Unit. The recommendations presented herein are based on this additional information and were used in the development of an economic benefit to cost ratio.

A-14.2 CRITERIA

A-14.2.1 Capacity Requirements

For the purpose of this report, the failure of floodwalls on piles is defined as a demand to capacity ratio. The floodwall's pile loading was determined using the Army Corps of Engineers CASE project Program CPGA with unfactored loads. This calculated load (demand) is compared to a mean pile capacity supplied by the geotechnical members of the team. A demand to capacity ratio of greater then 1.0 implies the pile foundation has ceased to function as designed.

A-14.2.2 Strength Requirements

For new structures designed with the Strength Design Method, loads are increased by multiplying service loads by appropriate load factors and nominal strengths are decreased by appropriate strength reduction factors. Load factors required by EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures* are a dead and live load factor of 1.7 and a hydraulic factor of 1.3. Combining these factors provides a total load factor of 2.2. The strength reduction factor for flexure, the typical controlling failure mechanism, is 0.90. Dividing the load factor by the strength reduction factor gives an overall factor of safety of about 2.45 for a new design.

Load and strength reduction factors were not used in the analysis of existing structures. This implies that if an existing structure has a calculated Factor of Safety of less then 1.0, the structure has ceased to function as designed. When considering an allowable factor of safety for existing structures, several allowable reductions can be taken into account. EM 1110-2-2104 allows for a 25% reduction in load for short duration loads with a low probability of occurrence, which would apply to flood events with a return period of greater than 300 years. A "performance" factor can also be applied to take into account the previous behavior of the existing structure. Knowing that the existing structure has performed well under loading and not shown visible signs of distress, it is assumed a 15% reduction in factored loads is acceptable. Combining the design factor with the frequency and performance factors produces an approximate 1.5 Factor of Safety for existing hydraulic structures in extreme loading conditions.

A-14.2.3 Uncertainty Analysis

For structures not meeting deterministic strength and stability criterion, a risk and uncertainty analysis was performed. A Taylor Series Method (TSM) of analysis was used in the calculation of structural risk and uncertainty. The TSM is appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in ETL 1110-2-547 (paragraph B-6.c). Examples of non-linear behavior for which the TSM should not be used include overturning stability analysis when the resultant is outside the kern of the base. Examples of degradation over time, which were not considered for the execution of this study, would include scour around piles, reactive concrete, sliding movement, and deteriorating drainage systems that affect uplift.

Risk Calculation. a. For strength calculations, uncertainty is measured by applying a mean and standard deviation to the concrete and steel strengths. The selected mean and normal standard deviation are based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

b. For stability calculations, uncertainty is considered by applying a mean and standard deviation to the soil unit weight and shear strength, and is based on values provided by the geotechnical engineers working on the study.

c. Failure is defined as the capacity to demand ratio (factor of safety) less than 1.0, or in other words, when the demand (loads) exceed the capacity (structural or geotechnical).

Material Properties. a. For the screening portion of the Kansas Citys Flood Damage Reduction Feasibility Study, the following structural properties will be used. The American Concrete Institute recommended the use of a 3,000 psi concrete strength around the 1940's and 1950's, the typical timeframe of construction for most of the levee structures in these feasibility studies. Limited design documentation and as-built drawings have been discovered that support the 3,000 psi original design strength assumption. For earlier concrete strengths, little information exists. It is currently assumed that 2000 psi concrete strengths are appropriate. If additional research information is discovered, this value will be updated.

b. Knowing the time period of construction (~1940's – 1950's) and based upon the Portland Cement Association's pamphlet *Engineered Concrete Structures*, 1997, an assumed reinforcing steel design yield strength, F_y, of 40 ksi is used for most computations, unless known or stated otherwise. This number has also been verified in the limited original design documents that have been found. For earlier structures (~1900's), the Concrete Reinforcing Steel Institute in *Engineering Data Report 48* suggests 33 ksi steel is typical.

c. Based on FEMA 310, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength. For reinforced concrete structures, Harr suggests a 14% standard deviation.

Concrete Strength Variation

1940's-1950's: $-\sigma = 3225$, $\mu = 3750$, $+\sigma = 4275$ (3000 psi min)

1900's-1920's: $-\sigma = 2150$, $\mu = 2500$, $+\sigma = 2850$ (2000 psi min)

Steel Strength Variation

1940's-1950's: $-\sigma = 43$, $\mu = 50$, $+\sigma = 57$ (40 ksi min)

1900's-1920's: $-\sigma = 35.5$, $\mu = 41.25$, $+\sigma = 47.0$ (33 ksi min)

A-14.3 FAIRFAX-JERSEY CREEK UNIT

A-14.3.1 Description of the Fairfax-Jersey Creek Unit - Structures

The Fairfax-Jersey Creek Unit is located on the left bank of the Kansas River from the Missouri Pacific Railroad Bridge (Kansas RM 0.3) downstream to the mouth of the Kansas River. It then extends along the right bank of the Missouri River from Missouri RM 367.5 to RM 373.9. Concrete capped sheet pile I-wall runs from station 2+58 to 28+51 along the southeast reach of the Fairfax-Jersey Creek Unit. An inverted-T cantilever floodwall on a pile foundation extends from station 287+86 to 302+32 along the upper northwestern reach of the Fairfax-Jersey Creek Unit. Four stoplog gaps, one sandbag gap, and 23 drainage structures are also located along the length of the unit.

A-14.3.2 Assumptions

Material properties could not be determined from existing documentation for a majority of the structures on the Argentine unit. As a result, estimated steel strengths, concrete strengths, and standard deviations were used for all strength analysis and risk computations for the structures on the Argentine unit (as noted in the previous section on uncertainty analysis).

Mean soil shear strengths and unit weights were assumed to be 28^{o} and 120 pcf respectively, based on the recommendations of the geotechnical engineers on the study team.

Before final design, reinforcing and concrete strengths shall be verified by testing.

A-14.3.3 Floodwall Analysis 287+86 to 302+32

Limited information was known about the pile size for the floodwall running from station 287+86 to 302+32 at the completion of the existing conditions portion of the study. Information found on the O&M Manual's as-built drawings only showed circular piles labeled as concrete piles with a minimum length of 20 feet. Based on this information, the known dimensions of piles for similar floodwalls built in the area around the same time period, and the walls performance in a substantial 1993 flood event, pile dimensions were assumed. Calculations revealed potential pile capacity and pile strength problems could exist with water near the top of the wall. It was determined that field tests would be required to determine the actual size and length of piles in order to more precisely determine the reliability of the floodwall. Subsequent field testing showed the piles to be 15.5 inch diameter, precast concrete piles penetrating 19 feet below the pile cap.

Pile Capacity Analysis. Using this updated data and unfactored loads in the Army Corps of Engineers CASE project Program CPGA, pile axial loads (demands) were determined for water at varying heights on the wall. These demands were then compared to the pile capacities (pile skin friction and end bearing) supplied by the geotechnical members of the study team. A Factor of Safety could then be calculated as the inverse of the demand to capacity ratio. Table A-14.1 summarizes the results for the critically loaded landside row of piles.

TABLE A-14.1 – Pile Capacities

Riverside Water Elevation	Landside Pile				
	Demand (kips)	Capacity (kips)	Demand to Capacity Ratio	Factor of Safety	Probability of Failure (%)
Top of Wall (ToW)	49.70	49.90	1.00	1.00	55.63
1 ft Below ToW	43.60	51.38	0.85	1.18	34.53
2 ft Below ToW	38.30	52.86	0.72	1.38	16.98
3 ft Below ToW	33.80	54.34	0.62	1.61	6.37
4 ft Below ToW	30.00	55.82	0.54	1.86	1.80

Pile Strength. No records have been found documenting the amount of reinforcing steel in the concrete precast piles. Since the piles are precast, they had to be transported; and reinforcing steel would need to be present for transportation. ACI recommends that 1% to 5% steel by volume is an acceptable range. Floodwalls of similar size and cross section built nearby in North Kansas City had 2% reinforcing steel in their concrete piles. It was assumed that the floodwall's piles in question contain six #9 bars, which gives slightly more then 3% steel. Using this assumed pile steel reinforcing, a combined axial and bending strength was determined for the piles. Combined axial and bending loads were determined from CPGA. Pile factors of safety for strength and probabilities of failure are summarized in Table A-14.2.

TABLE A-14.2 – Pile Strength

Riverside Water Elevation	Pile Strength Factor of Safety (> 1.5 Req'd)	Probability of Failure (%)
Top of Wall (ToW)	0.84	99.97%
1 ft Below ToW	0.92	95.8%
2 ft Below ToW	1.01	42.6%
3 ft Below ToW	1.12	1.0%
4 ft Below ToW	1.26	0%

During the 1993 flood event, water crested approximately 3 feet below the top of wall. No visible deformations of the wall were observed. This would further validate the pile reinforcing assumption of approximately 3%. For the 1993 flood event, the pile strength had a factor of safety greater than 1.0 and a probability of failure of about 1%.

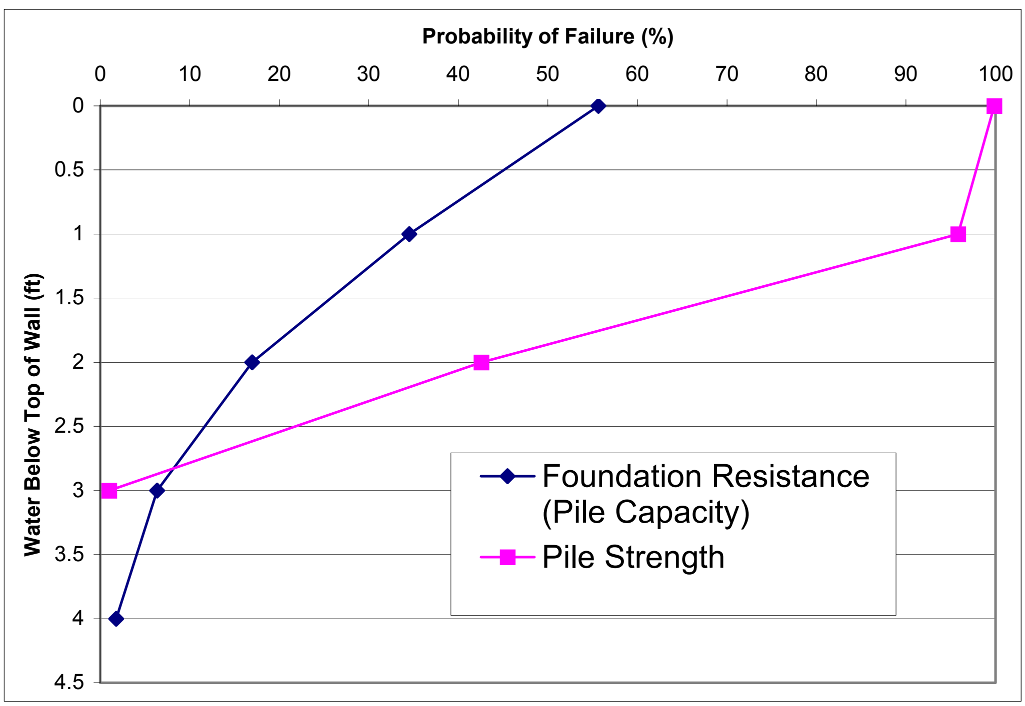
Floodwall Strength. Flexure and shear strengths for the floodwall stem and pile cap were computed based on an assumed reinforcing steel yield strength of 40 ksi and a concrete compressive strength of 3.75 ksi. A comparison of required strength to actual strength showed the foundation flexural steel to control. Results are summarized in Table A-14.3. The floodwall stem and pile cap were determined to be adequate for all water elevations.

TABLE A-14.3 – Floodwall Strength

Riverside Water Elevation	Floodwall Strength Factor of Safety (> 1.5 Req`d)
Top of Wall (ToW)	2.4
1 ft Below ToW	2.8
2 ft Below ToW	3.4

Conclusions. Exhibit A-14.1 summarizes the probability of failure for the BPU floodwall based on both pile strength and foundation resistance (pile capacity). Because of the insufficient pile strengths and pile capacities, foundation modifications or floodwall replacement is required to reduce the load on the existing piles and achieve an acceptable factor of safety.

EXHIBIT A-14.1 BPU Floodwall Probability of Failure

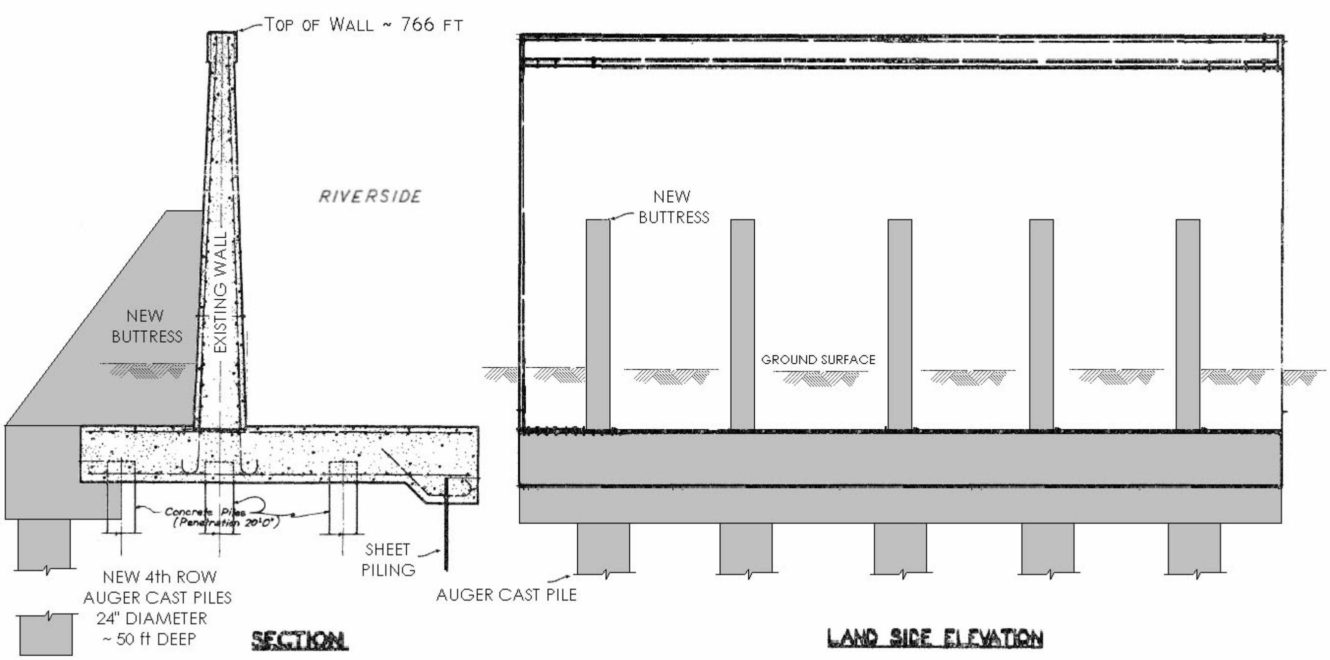


A-14.3.4 Floodwall Modifications 287+86 to 302+32. A variety of alternatives, including an additional fourth row of landside piles, placing a new floodwall behind the existing wall, and soil modification using a pressure injected grout were considered to prevent excessive loading of the landside pile.

Modified Wall. As mentioned, one possible solution is the construction of an additional fourth row of auger cast piles landside of the exiting walls. The piles would be auger cast and not driven because of the close proximity of the Kansas City Board of Public Utilities power plant and the sensitive nature of much of the equipment used for their operations. For the purposes of the cost estimate, 50-ft deep, 24" diameter auger cast piles at seven foot on center are assumed sufficient to relieve the overloading on the landside pile. For plans and specifications, a more thorough design would need to be performed to size the auger cast piles.

The fourth row would be joined to the existing pile cap by doweling into the existing floodwall pile cap. A buttress would be installed over each pile to transfer load from the wall to the additional piles. The existing stem wall's longitudinal steel has been checked to verify there is sufficient longitudinal reinforcing to transfer the wall loading to the buttresses. Exhibit A-14.2 illustrates the purposed wall modification.

EXHIBIT A-14.2- Floodwall Modification



Replacement Wall. In some reaches it would be possible to construct a new floodwall behind the existing wall. The cost of a new floodwall on auger cast piles was compared to that of the proposed floodwall modification with an additional forth row of piles. Taking into consideration the additional costs of joining the new and existing walls and cutoff sheet piling, however, negates the cost advantages of the new wall. As a result, the modified wall was determined to be the desired alternative. For a comparison of the two possible alternatives, see Exhibits A-14.3 and A-14.4 in the Supplemental Exhibits section.

A-14.3.5 Stop Log Gap

Based upon the concerns and recommendations of the Fairfax Drainage District, the 17.5-foot wide, 11-foot tall stop log gap on spread footings at Station 312+74.6 was investigated for possible strength and stability deficiencies. Results are summarized in Table A-14.4.

TABLE A-14.4 – Stop Log Analysis

Criteria	Required	Water at Top of Gap	Water at 1.8 ft Below Top of Gap
Sliding Stability	> 1.3 Factor of Safety	1.2	1.6
Rotational Stability	> 25% Base in Compression	100%	100%
Bearing Pressure	< 150% Increase in Allowable Bearing Pressure	135% Increase	130% Increase
Strength	> 1.5 Factor of Safety for Existing Structure	1.4	2.2
Probability of Failure (Strength)	----	1.2%	~0%

Analysis shows a minimal chance of failure when water is at the top of the stop log structure. Based on the water surface profile provided by the hydrology and hydraulics members of the study team, water will be approximately 1.8 ft below the top of the stop log gap when water begins to overtop the lower end of the Fairfax-Jersey Creek Unit. At this water surface elevation, the stop log gap performs adequately and is therefore considered acceptable.

A-14.3.6 Controlling Mechanism

As the Missouri River rises along the Fairfax-Jersey Creek Unit, overtopping first occurs at the lower end of the unit, around the Jersey Creek Tieback located under the Lewis and Clark Viaduct. At the time of this initial downstream overtopping, water is approximately 2.5 feet below the top of the Fairfax-Jersey Creek BPU Floodwall, creating a 22% probability of floodwall failure. Flood fighting and temporary low-level berms along the lowest portions of the Jersey Creek Tieback are assumed as a reasonable short-term overtopping remedy. Longer-term permanent solutions will be examined in the next phase, subsequent to the interim appendix of this feasibility study.

Given that flood fighting and/or a more permanent solution is put in-place during a flood, the floodwaters would then seek the next lowest point in the line of protection which occurs at the Jersey Creek Outlet Structure. As this outlet is overtopped, water is approximately 1.7 feet below the top of the BPU floodwall, creating a 59% probability of floodwall failure. Sandbagging and flood fighting at the outlet is considered an adequate remedy for the short and long-term as the low spot is less than two feet below the prevailing lower-end elevation.

Eventually, as floodwaters continue to rise, even with the aforementioned measures, floodwater will overtop the entire lower end of the unit. At the time of general lower-end overtopping, water is approximately 1-foot below the top of the BPU floodwall. This condition creates a 96% probability of floodwall failure.

The sequence of overtopping points provides an opportunity for strengthening the floodwall and removing a potential floodwall failure from the likely failure sequence as water reaches the last 3 feet of height on the Fairfax-Jersey Creek Unit BPU floodwall. We thus compute the economic benefits for strengthening the floodwall and ensuring that general lower-end overtopping is the most likely failure scenario. Exhibit A14-5 summarizes the overall probabilities of failure. Strengthening the wall also has the added benefit of ensuring that a catastrophic upper-end overtopping (from floodwall failure) does not occur. Overtopping at the upstream end of a unit can be especially damaging as high-velocity floodwaters blow through the point of failure and cascade down through the unit, creating more severe damage than a gradual lower-end overtopping scenario. The economic analysis cannot really capture this increased devastation, but it is real and worthy of note.

It should be noted that as part of the total Interim Engineering Appendix to the Feasibility Report recommended plan for the Fairfax-Jersey Creek Unit:

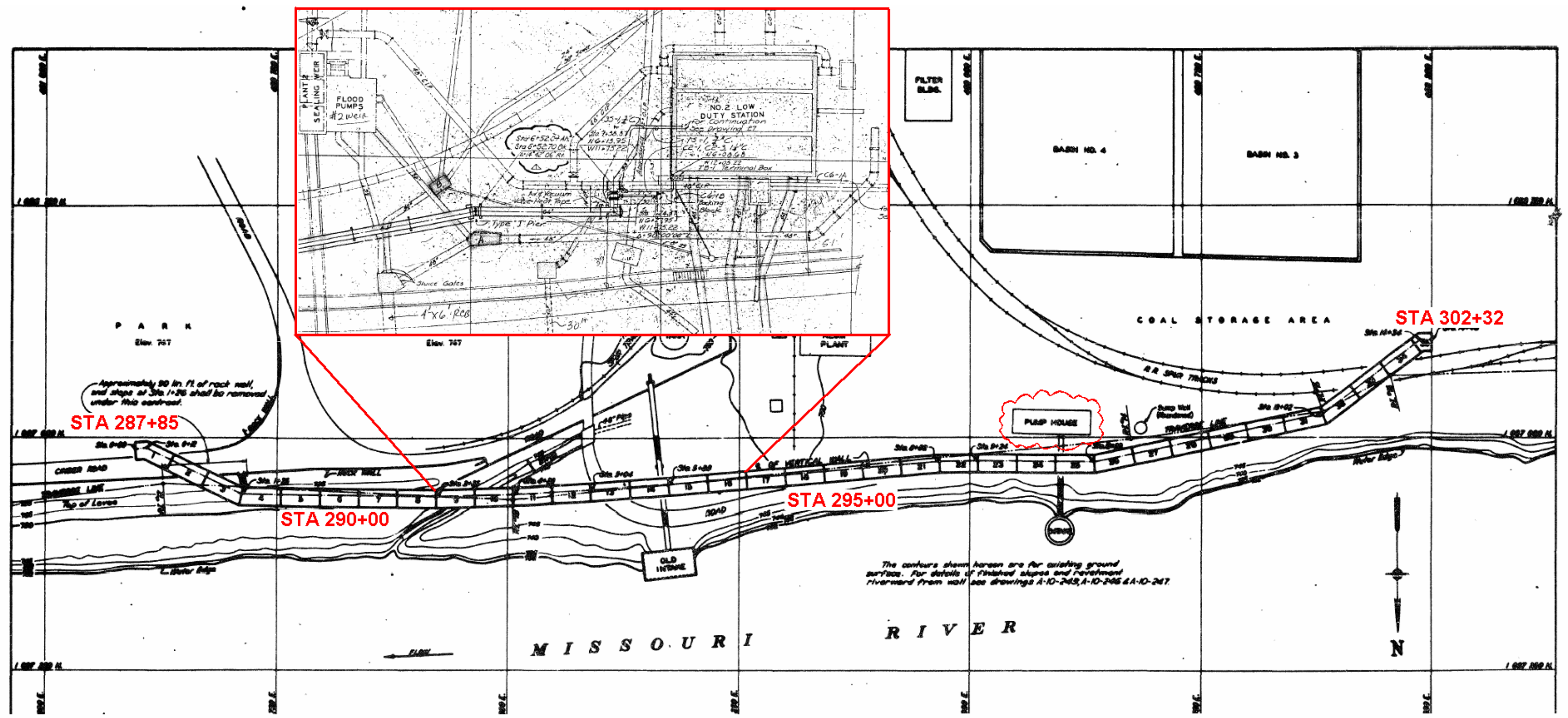
- Reconstruction of the Fairfax-Jersey Creek sheet pile wall (addressed in the Geotechnical Analysis – Fairfax-Jersey Creek (Sheetpile Wall) chapter) has proven to be economically viable as a stand-alone (independent) remedy. Thus, the sheetpile wall reliability is not a part of the Fairfax-Jersey Creek Unit BPU floodwall failure analysis.
- As part of the total work package for the Fairfax-Jersey Creek Unit, the Kaw Valley Drainage District and the Unified Government of Wyandotte County will undertake locally funded ("KCK-UG Public Levee" area) municipal wharf improvements germane to the line of protection.

A-14.4 REFERENCES

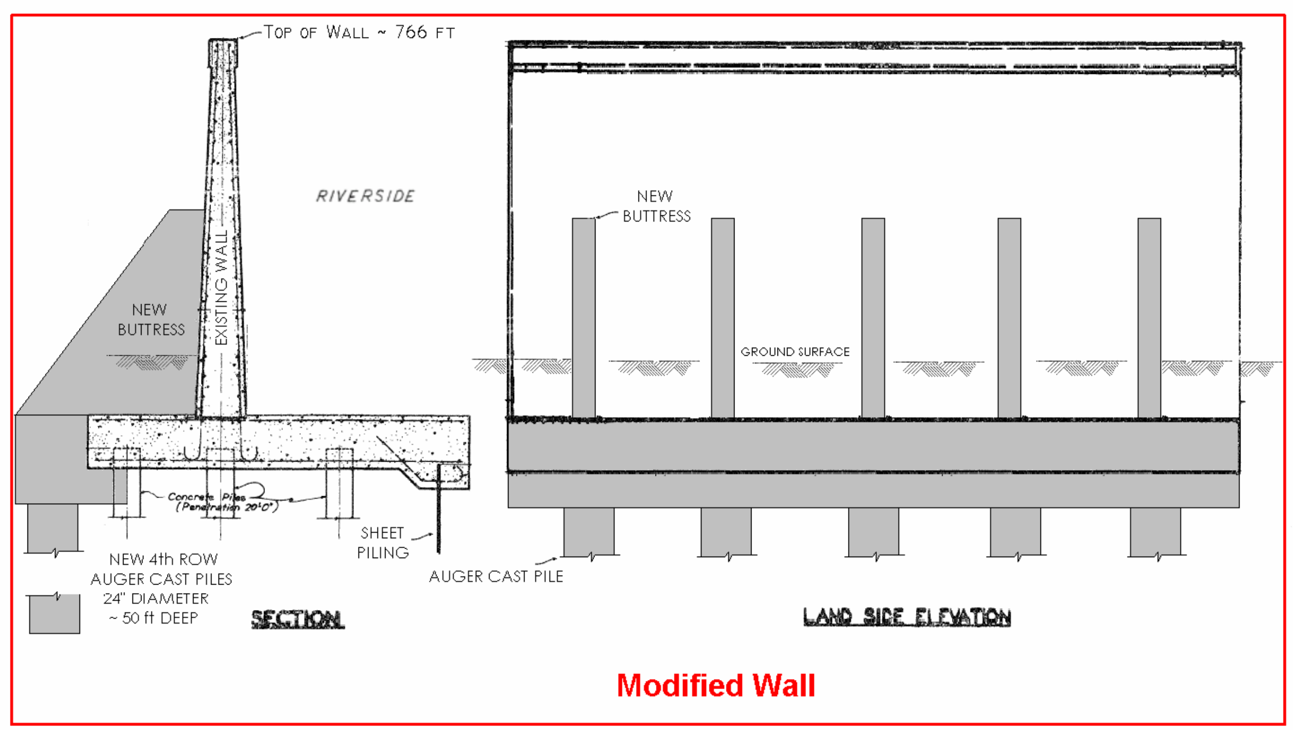
1. US Army Corps of Engineers (1999), *Reconnaissance Report – Kansas Citys, Missouri and Kansas Flood Damage Reduction Project*, Kansas City District.

A-14.5 SUPPLEMENTAL EXHIBITS

EXHIBIT A-14.3 –Modified Floodwall Alternative



Current plan is to leave adandoned pump house intact, basement to be filled, and pipes grouted.



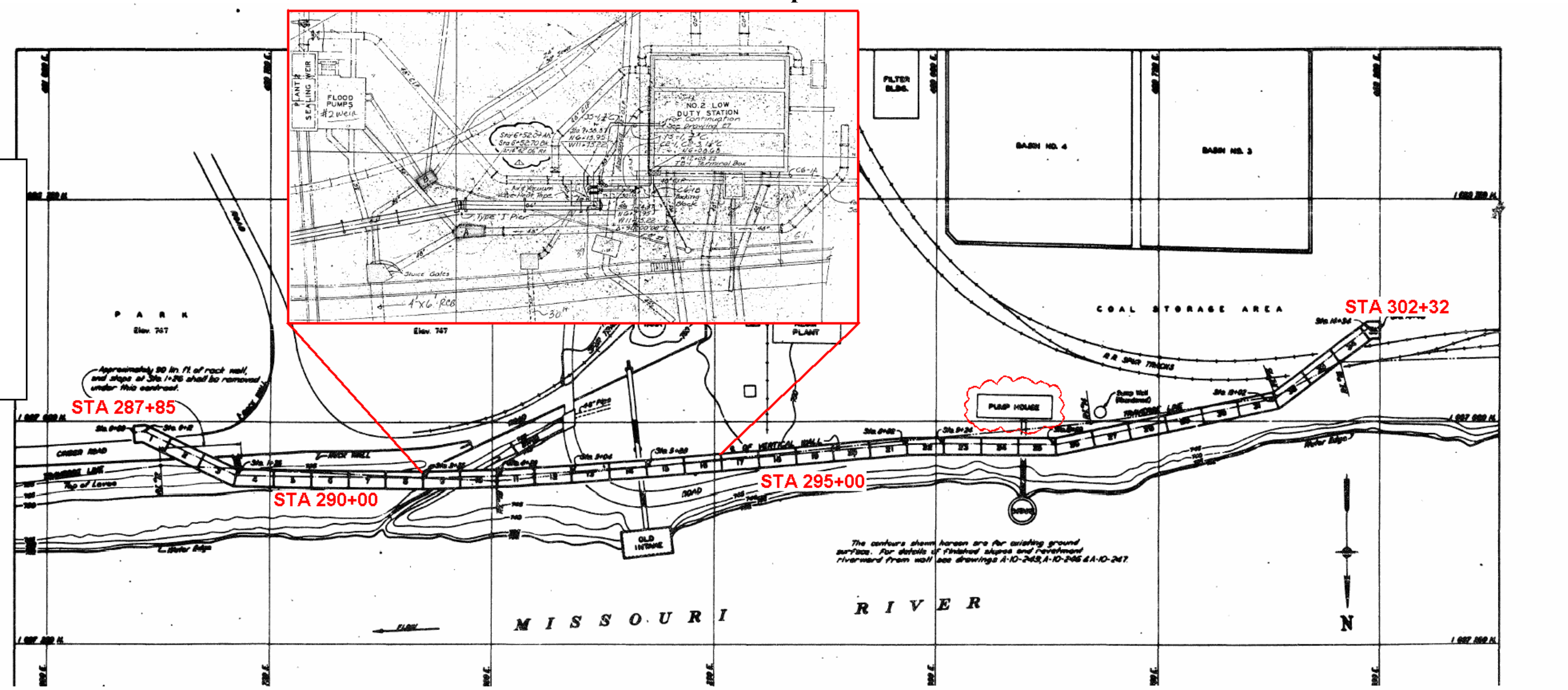
ALTERNATIVE 1

MODIFIED FLOODWALL

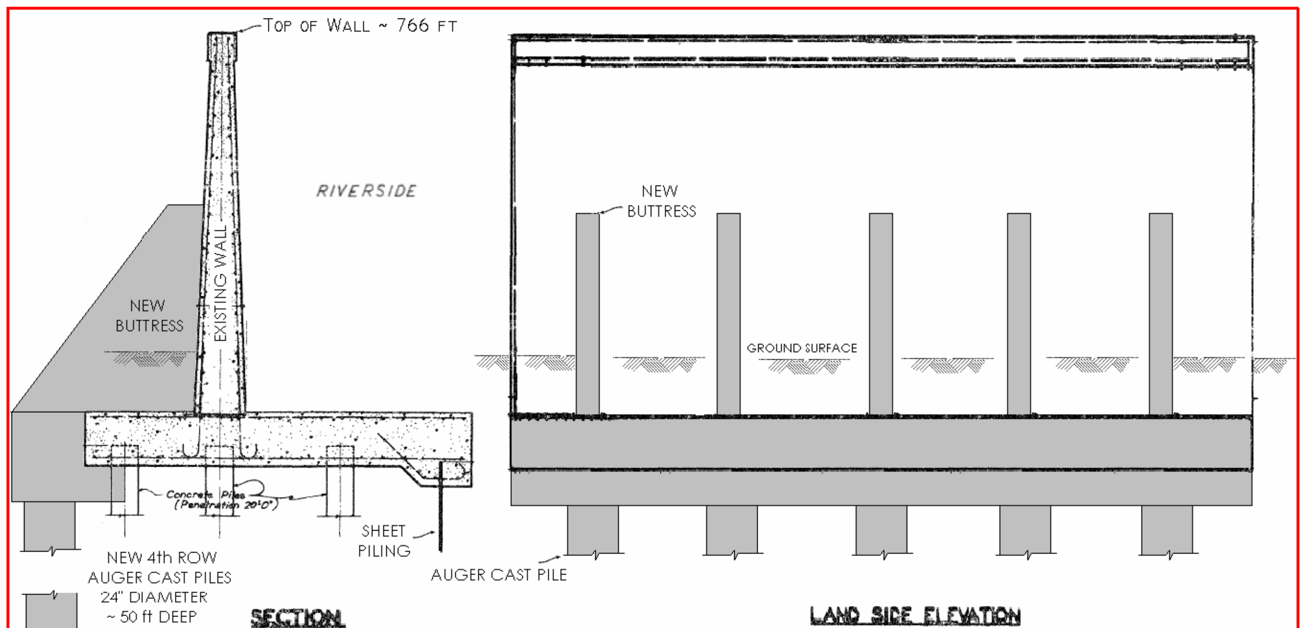
EXHIBIT A-14.4 – Combined Modified and Replacement Floodwall Alternative

ALTERNATIVE 2

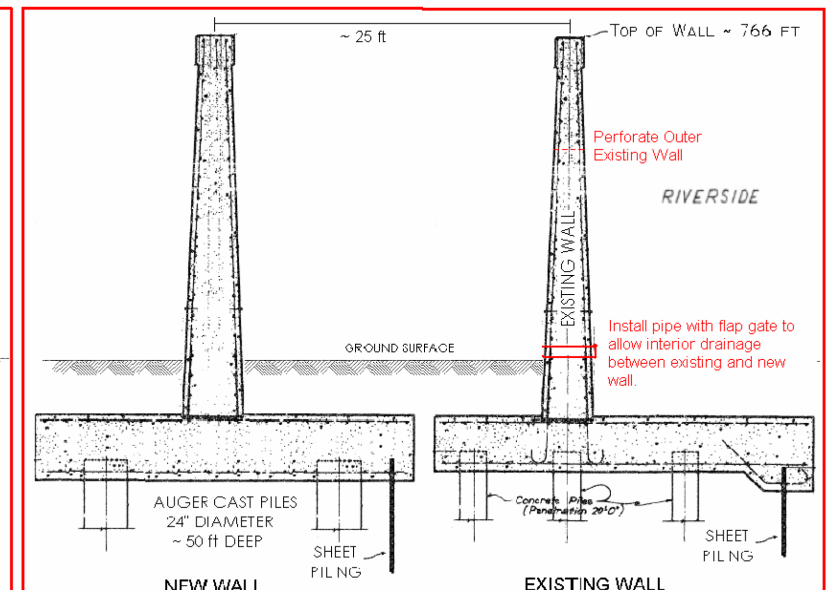
COMBINATION FLOODWALL:
NEW FLOODWALL &
MODIFIED FLOODWALL



Current plan is to leave adandoned pump house intact, basement to be filled, and pipes grouted.



Modified Wall Segment ~ Station 290+00 to 295+00



New Wall Segments
~ Stations 287+85 to 290+00 & 295+00 to 302+32

EXHIBIT A14-5 Fairfax Drainage District Probabilities of Failure